

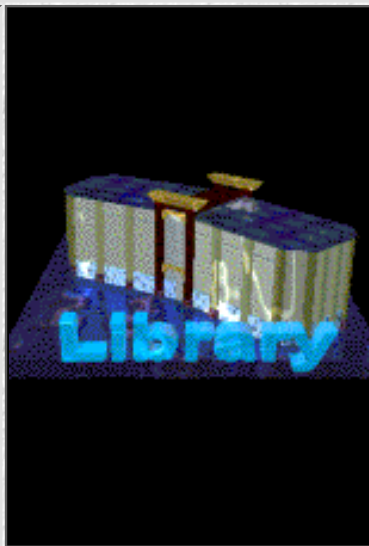


# Addendum to Highways in the River Environment/ Hydraulic and Environmental Design Considerations

**HEC 16**  
**July 1980**

Welcome to  
HEC  
16-Addendum  
to Highways in  
the River  
Environment/  
Hydraulic and  
Environmental  
Design  
Considerations

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The purpose of this addendum is to supplement the publication, "Highways in the River Environment--Hydraulic and Environmental Design Considerations" (HEDC), prepared for the U.S. Department of Transportation, Federal Highway Administration, in 1975. (Available from U.S. Government Printing Office, Washington, D.C. 20402, Catalog Number TD 2.8:H53/2) It is designed to provide guidance at two levels, for those engineers who must make decisions based on reconnaissance types of data and for engineers preparing and reviewing final designs.

The objective of this supplement is to present, in a concise format, significant river mechanics principles and useful design equations or parametric relationships, and to demonstrate their application. It is assumed the user of this manual is trained in open channel hydraulics and is an experienced hydraulics engineer.

The information presented in the first six sections of this addendum is generally qualitative. The subjects discussed include sources of data, preliminary design considerations, classification of streams, evaluation of channel stability, prediction of channel movement, determination of the potential for the channel to change from one type to another in response to channel modifications, and estimation of the range of channel roughness. The remaining sections deal with specific computation techniques for designing meandering channels, estimating sediment transport, designing meanders for new channels, estimating scour depth in bends and at piers, and reviewing the safety factor for riprap designs.

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This document was prepared by Dr. F. J. Watts, Department of Civil Engineering, University of Idaho, under contract with the Federal Highway Administration (FHWA), Office of Development, Implementation Division. The guidance, assistance, and review of material by Mr. Daniel S. O'Connor (former contract manager) and Mr. Robert D. Thomas, contract manager, FHWA, and the help of Mr. Muhammad S. Mir, Graduate Assistant, and Mrs. Helen D. Luker, Technical Typist, University of Idaho, are gratefully acknowledged. The FHWA, Office of Engineering, Bridge Division, Hydraulics Branch prepared the document for publication.

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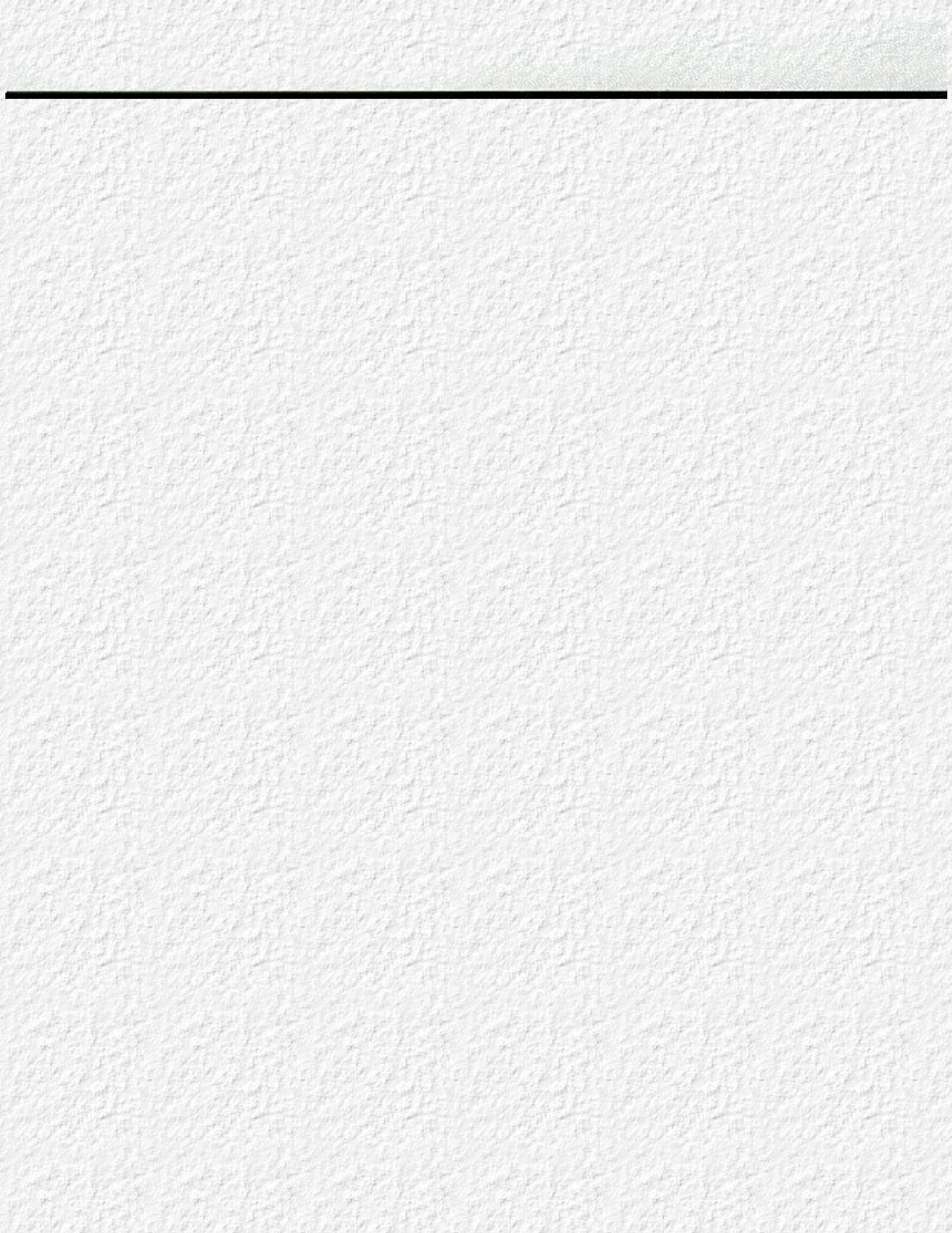
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<u>To Convert</u>	<u>To</u>	<u>Multiply By</u>
inches (in)	millimetres (mm)	25.40
inches (in)	centimetres (cm)	2.540
inches (in)	metres (m)	0.0254
feet (ft)	metres (m)	0.305
miles (mi)	kilometres (km)	1.61
yards (yd)	metres (m)	0.91
square feet (ft <sup>2</sup> )	square metres (m <sup>2</sup> )	0.093
square yards (yd <sup>2</sup> )	square metres (m <sup>2</sup> )	0.836
acres (acre)	square metres (m <sup>2</sup> )	4047.
square miles (mi <sup>2</sup> )	square kilometres (km <sup>2</sup> )	2.59
cubic feet (ft <sup>3</sup> )	cubic metres (m <sup>3</sup> )	0.028
pounds (lb)	kilograms (kg)	0.453
tons (ton)	kilograms (kg)	907.2
pounds per square foot (psf)	newtons per square metre (N/m <sup>2</sup> )	47.9
pounds per square inch (psi)	kilonewtons per square metre (kN/m <sup>2</sup> )	6.9
gallons (gal)	litre (L <sup>3</sup> )	3.8
acre-feet (acre-ft)	cubic metres (m <sup>3</sup> )	1233.
gallons per minute (gpm)	cubic metres/minute (m <sup>3</sup> /min)	0.0038







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**English to SI (Metric) Conversion Factors**

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# Chapter 1 : HEC 16

## Introduction

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The major objective of designing a highway river crossing is to develop a cost effective combination of structures and embankments which satisfies projected traffic requirements with a minimal adverse impact on the channel system and its environment.

An alluvial channel is composed of material transported by the river. Depending upon the magnitude and duration of flow, the river continuously changes position and shape as a consequence of hydraulic forces acting on its bed and banks. The banks will erode, sediment will be deposited on the flood plain and islands, and side channels will change with the passage of time. Material presented in this addendum will be limited to alluvial channels and those channels that display predominant characteristics of alluvial channels. An alluvial channel system can be in a tenuous temporarily balanced condition, henceforth referred to as threshold, where only a minor disturbance of the channel can cause significant channel changes. Usually these changes can be tolerated; in other instances they can be catastrophic. One purpose of this manual is to call to the attention of the planner and designer some tools for analyses which can be used to evaluate the likelihood of significant changes occurring.

Because of the numerous independent variables, wide range of geomorphic characteristics, and general nonuniformity of channel materials, it is risky to generalize river mechanics. Therefore, information presented in this manual must be used judiciously. There is no substitute for experience and judgment where decisions must be made and there are distinct limitations on all of the design aids presented. In all instances, the characteristics and response of similar streams in the region to similar modifications should be considered. It is assumed that the user has carefully studied the HEDC manual and realizes that each river system is unique, each problem is different, and engineering judgment must be applied in all instances.

In many instances, decisions will have to be made without adequate data and where uncertainty prevails. A thorough understanding of the principles outlined in HEDC will provide a firmer basis for decision under these circumstances. Included in the Summary section is a discussion of the analysis techniques found in HEDC for site-specific problems and a suggested procedure for the engineering analysis of proposed projects.

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# Chapter 2 : HEC 16

## Preliminary Design Considerations

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The types and amounts of data needed for planning, designing, and evaluating the impact of river crossings and lateral encroachments vary from project to project depending upon the class of proposed highway, type of river, geographic area, the severity or degree of encroachment, traffic volumes, risk of loss, costs, and numerous other considerations.

The data, preliminary calculations, and the analyses of alternative routes should be documented in a report. Such a report serves to guide the detailed designs and also provides reference material for environmental impact analysis and documentation for any litigation which may arise.

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### **Data Needs and Sources (VII-1 to VII-15)<sup>1</sup>**

Typical data or information required include: area maps, vicinity maps, site maps, aerial and other photographs, hydrologic data, geologic maps, soil survey data, and environmental data. Auxiliary data such as climatological data and hydraulic data may also be required. Checklists for data and sources of data are discussed in Chapter VII, HEDC.

Personnel involved in preliminary site selection should inspect potential sites prior to and during site analysis. Those responsible for final plans must also inspect the site. A local maintenance superintendent with knowledge of the history of the stability of rivers in the region should be a member of these inspection teams.

<sup>1</sup> Roman numerals in parentheses and page and chapter numbers throughout the text refer to HEDC where additional information on the subject can be found.

---

### **Considerations in Risk Analysis**

Designs for all stream crossings and highway encroachments on streams should be predicated on a rational decision-making procedure. This entails evaluation of alternative locations and designs to develop cost-effective solutions. Construction costs which can be justified by future savings in reconstruction and repair costs, costs for traffic delay and indirection, and property losses represent a savings in public funds and a judicious use of highway construction and maintenance funds.

This addendum does not address the economic analysis which should go into decisionmaking or the evaluation of alternative designs. It does, however, provide information which can be



used to develop practicable alternatives and a basis for evaluating the probable success of each.

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## History of Stream Stability

Historical information is qualitative and must be obtained from longtime residents and maintenance personnel, agency records, newspaper archives, or by comparing old aerial photographs with recent photographs. Topics of concern are:

1. Is the river as it now exists relatively stable, i.e. has the position (alignment) of the channel changed significantly during the previous span of years equal in number to the proposed design life of the new structure? Is the floodway aggrading, degrading or is it essentially stable?
2. If instability exists, have there been significant modifications in upstream land use (construction of reservoirs, logging, large paved expanses, etc.) which significantly altered discharge or sediment transport over the years? Is a major change in land use planned during the design life of the proposed construction?
3. Has the channel been subjected to a flood equivalent to the design flood during this span of years? If so, what were the consequences?
4. What effects will the proposed construction have on the stream? Will the construction result in increased velocity and/or redirection of flow which may trigger erosion and result in an unstable system? Will the crossing or encroachment significantly modify the floodway such that channel instabilities will be initiated?

The extent of analysis required will depend on the answers to these questions, the cost of damage associated with predicted channel change, and the scope and design life of the project.

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## Classification of Streams (I-1, I-14)

Classification of streams is useful in that some types of streams are less stable than others and thus will require more detailed analysis. Three of the most significant characteristics from a stability viewpoint are active flood plains, dominant mode of sediment transport, and sinuosity.

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### Active Flood Plains

Rivers with active flood plains require special consideration. An active flood plain conveys a significant amount of water, approximately more than 20 percent of the flow during a 15-year flood. This is in contrast with geologically old remnant flood

plains adjacent to rivers that may provide storage but very little overland flow.

The practice of constructing fill over active flood plains and forcing the flood through a contracted main channel creates many problems and should be given careful consideration. Problems which must be considered when contractions are necessary include: aggradation of the upstream channel and flood plain, higher flood stages upstream of the encroachment, and high velocity flow and consequent erosion around piers and abutments.

---

## **Mode of Sediment Transport**

If the dominant mode of sediment transport is by bedload, there will be gross changes in Manning's "n" and stage with variation in discharge and, to some extent, temperature. During severe flood events, particularly in braided streams, the bed material may accumulate in large bars forcing the thalweg to shift laterally creating a new channel. At contracted sections, local upstream aggradation on the order of several feet can occur. If the dominant mode of sediment transport is by suspended load with a minimum amount of bedload, even during high flow, and if a significant amount of the suspended material is clay, the channel will generally be well incised and relatively stable. The banks will be steep and well sealed with clay. Widening of the channel occurs as the result of slumping of saturated banks after the flood wave recedes. The slumped material along the bank generally revegetates causing some obstruction to flow. However, the channel will be wider as a result of slumping and may provide an equivalent floodway.

---

## **Sinuosity**

Sinuosity is defined as the ratio of the length of the river thalweg to the length of the valley proper. The thalweg is the path of deepest flow. Based on sinuosity and information shown on Figure 1.2.1, page I-3, a river can be classified as straight, braided, or meandering. Rivers with sinuosity less than 1.5 are usually considered straight.

Straight alluvial channels are not common. If a straight channel is formed in alluvial material, riprap or bank protection will generally be required to maintain the straight alignment.

Braided rivers are generally unstable. Significant lateral movement of the main channels may occur with each large flood. Measures which have been used in anticipation of channel shifting include: adding extra spans to the ends of a bridge structure, deep placement of footings in anticipation of shifting thalweg, hardening the banks (riprap), and forming a funnel with riprapped berms at the upstream side of the bridge. The upstream opening of the transition must be sufficiently wide to insure that the ends will not be flanked. Spur dikes can also be used for this purpose.



Well incised meandering rivers are generally the most stable. The extent of channel movement with time can be determined by examining aerial photographs taken at different times over a period of several years.

Additional information concerning straight, braided, and meandering channels can be found on pages IV-4 through IV-14 and in chapter V.

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# Chapter 3 : HEC 16

## Stability of Channels - Lane Relationship

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The Lane relationship (IV-18) is useful for evaluating the qualitative channel responses to gross changes in either slope, discharge, bed material transport or mean size of bed material.

Lane Relationship:  $QS \approx Q_s D_{50}$

Where:

- Q = Average discharge in the channel.
- S = Slope of the channel.
- $Q_s$  = Bed material transport.
- $D_{50}$  = Fall diameter of bed material.

From a practical viewpoint,  $D_{50}$  (median diameter) as defined by sieving or other mechanical means, can be used in lieu of fall diameter.

The Lane equation states that if one of the four variables changes significantly and two of the other variables remain essentially unchanged, the fourth variable must change over a period of time.

Example: With reference to case 8, table 8.2.1, page VIII-6, or Figure 4.4.2, page IV-19,  $Q_s$  decreases as sediment is trapped in the reservoir and Q and  $D_{50}$  are initially assumed to remain unchanged; therefore, the slope, S, must decrease if the Lane equation is to remain in equilibrium.

The clear water release from the dam will entrain sediment, the bed will degrade and, with this removal of sediment,  $D_{50}$  will increase somewhat as a result of armoring. The net result is degradation downstream of the dam and a decrease in channel slope below the reservoir.

Example: With reference to Figure 4.4.1, page IV-18,  $Q_s$  increases,  $D_{50}$  and Q initially remain constant; therefore, S must increase if the Lane relationship is to remain in balance.

The 13 hypothetical cases of river environments and their response to changes within the floodway (VIII-3 to VIII-14) should be carefully studied by both planners and design engineers. Often, sites can be eliminated early in the planning stage if gross channel changes can be anticipated to result from the proposed construction. If other circumstances dictate that the crossing must be constructed at a difficult site, these illustrations are useful for identifying a qualitative response.

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# Chapter 4 : HEC 16

## Lateral Channel Shifts

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Some useful techniques for qualitative evaluation of the stability of channels (not specifically covered in HEDC) are presented below.

### Incised Channels

Data required:

1. A series of aerial photographs, each set for a different year, the series extending over as long a period as possible. See page VII-8 for sources of aerial photographs.
2. Flow data for the stream in question obtained from water supply papers, U.S. Department of Interior, Geological Survey, Water Resources Division, or from State or local records.

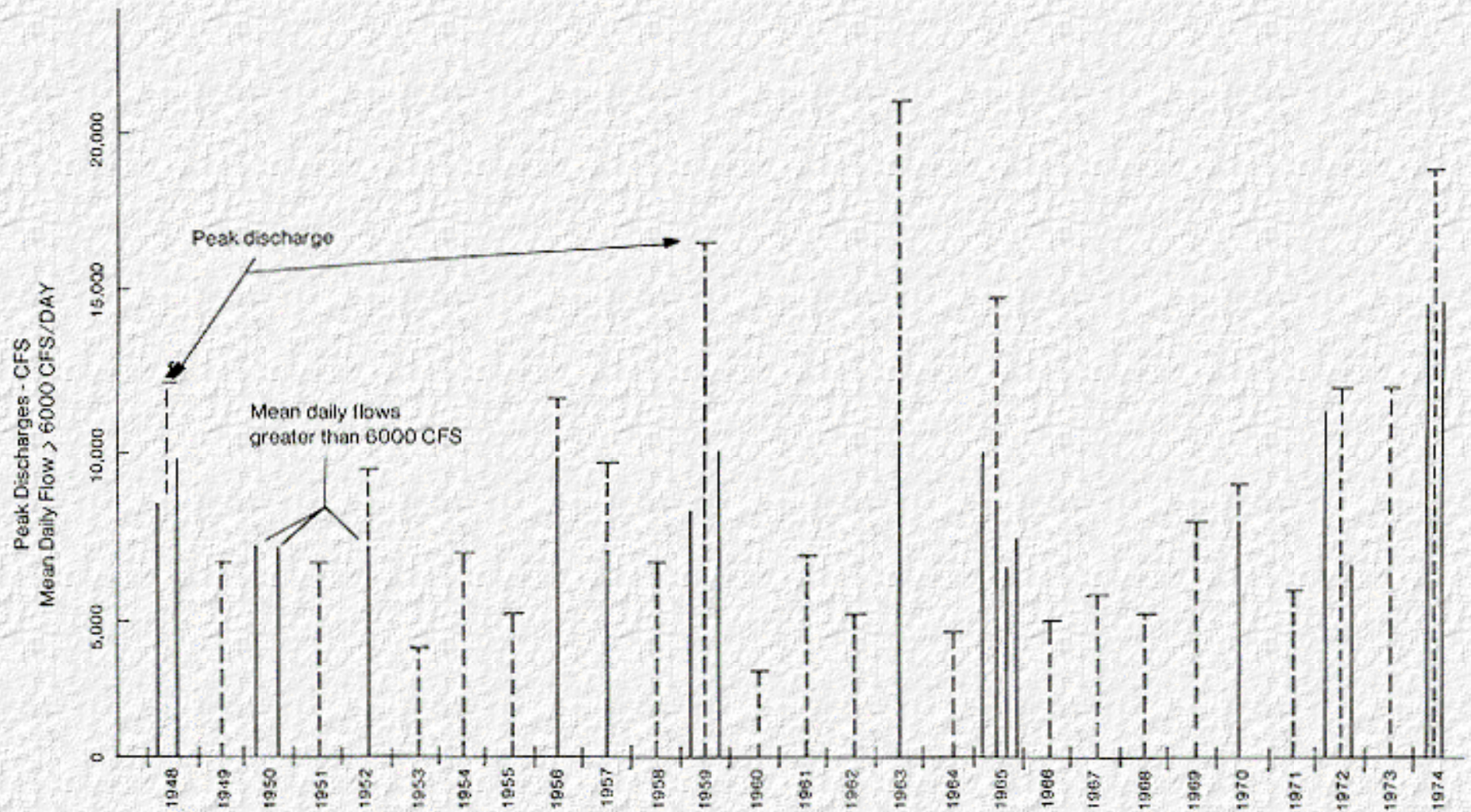
The series of aerial photographs can be used conjunctively with a plot of time versus peak flow and discharge-days for Q-dominant to review the history of channel stability. Q-dominant is sometimes referred to as the formative discharge (IV-24, IV-25, V-25) and in this example is the discharge where significant channel shaping occurs. Changes in channel movement noted between two successive sets of aerial photographs can generally be attributed to discharge activity or a sequence of events which can be identified on this plot. Bank-full discharge is often selected as Q-dominant where the banks of the channel are uniform. However, each site must be examined and, depending on the strata of the banks, a different criteria for selecting Q-dominant may be used.

A channel may be stable for many years, then a sequence of high peak flow years, sometimes accompanied by long periods of high cfs-days, will trigger significant changes.

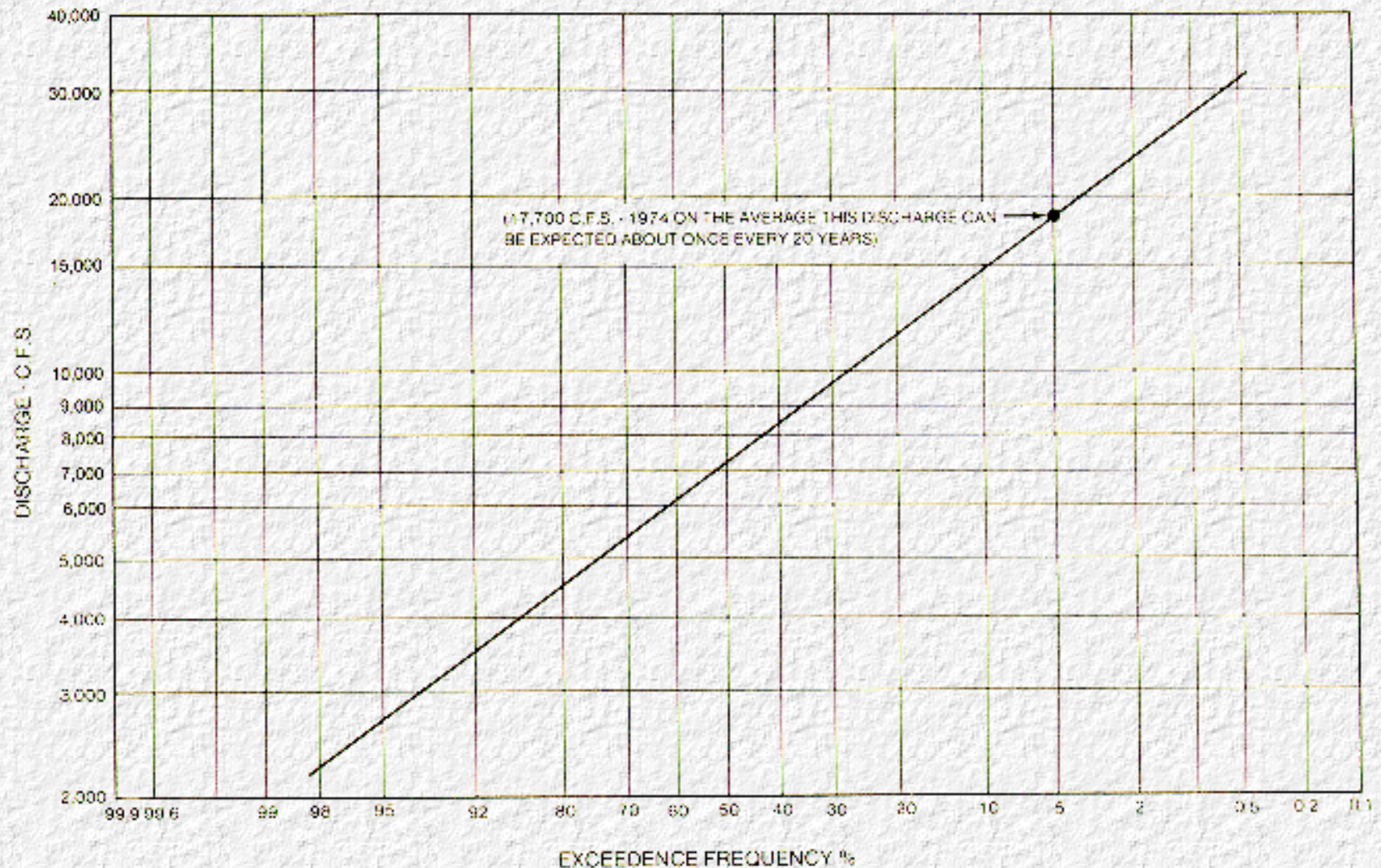


*Example:* [Figure 1](#) is an example of a plot of peak flows and Q-dominant, flows greater than 6000 cfs-day, plotted over a period of 26 years. [Figure 2](#) is a standard flood frequency curve for the same river. For the period 1948 to 1963, information shown on aerial photographs indicates that the channel was stable, meandering, narrow and incised with good vegetative growth on both banks. The flood of record occurred in 1963. Aerial photographs taken shortly after the 1963 event show many of the banks were denuded and left in an over-steepened condition. These over-steepened banks slumped over the next 2 years. High sustained flows in 1965 cleared out some of the slumped bank material and again left the banks in an over-steepened, stripped condition. Aerial photographs and personal interviews indicate that the channel was reasonably stable for the period between 1965 and 1972 with some bank scour in 1972 and 1973. In 1974, a 20-year peak accompanied by two consecutive days of the highest mean daily flows (discharges were nearly 30 percent higher than any individual mean daily flow on record) resulted in considerable bank erosion. The eroded material, transported primarily as bed material, temporarily raised the stream bed by 2 to 4 feet as determined by soundings at a USGS gaging station in the lower portion of the reach. The stage in this reach, considering the 2 to 4 feet of temporary aggradation, was about the same as a stage associated with a 100-year peak event assuming minimum bed activity.





**Figure 1. Chronological Sequence of Peak Discharge and Mean Daily Flows Greater Than 6,000 CFS (Dominant Discharge Assumed To Be 6,000 CFS)**



**Figure 2. Flood Frequency Curve**

Major changes in alignment occurred where convoluted oxbows were scoured away and points of land were cut off. This resulted in a shortened thalweg and, coupled with three upstream cutoffs which have occurred in the past 30 to 40 years, left the channel overly steep and in a very active state. A series of aerial photographs starting in 1958 clearly display the changes in channel geometry which have occurred.

If a crossing is necessary in this active channel, the banks must be stabilized in the vicinity of the structure and adequate vertical clearance provided for high flows, large dunes, and temporary bars which result when the



channel is overloaded with slumped bank material and debris. Where a significant increase in bed elevation is anticipated, care must be taken that the entire preconstruction channel cross section, or even a slightly larger cross section, is available for peak flows.

By examining the Flood Frequency-Maximum Peak Discharge curve ([Figure 2](#)), it can be verified that the peak flow experienced in 1974 (17,700 cfs) was not a rare event, probably an event with a 20-year recurrence period. Thus, the maximum annual peak discharge was not the significant variable. The chronological sequence of mean daily flows associated with Q-dominant, 6000 cfs-day in this case, is the most significant data.

The critical stage associated with Q-dominant was selected after studying the stratigraphy of the banks and noting the lower portion contained a stable clay layer with very little lateral scour occurring until the water rose to the sand-silt layers. This interface was selected as the critical stage, and then Q-dominant associated with this stage was estimated.

Other excellent examples of the use of aerial photographs for the hydraulic design of crossings are shown in "Guide to Bridge Hydraulics."<sup>2</sup>

<sup>2</sup> C. R. Neill, "Guide to Bridge Hydraulics," Roads and Transportation Association of Canada, University of Canada, University of Toronto Press, 1973.

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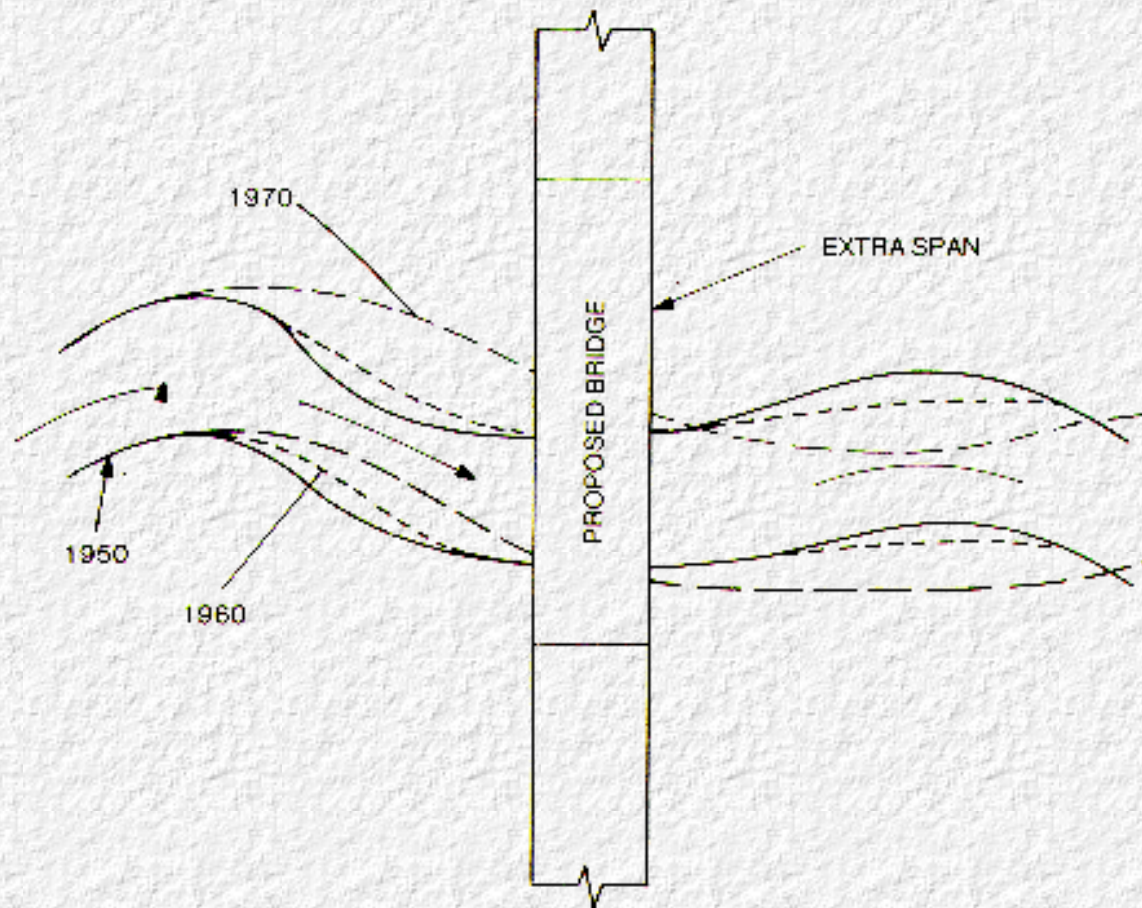
## Meandering Surface Bends

Where channels are not so well incised (free or meandering surface bends) (V-7, V-8), aerial photographs can be used for estimating migration.

*Example:* [Figure 3](#) is a composite plot showing movement of a channel over a period of years. This drawing was constructed by:

1. Reducing all aerial photographs of the specific reach to a common scale. The reduction was verified by checking scaled distances between common objects located near the channel. The channel is nearly horizontal so there is no gross distortion.
2. Using transparent paper to trace the outline of the banks of the channel for each set of aerial photographs using different colors or different line styles for each set of photographs.





**Figure 3. Composite Plot of Meanders Adjacent to Proposed Bridge Site**

It is apparent from [Figure 3](#) that the meander will be moving under the bridge if the channel continues to change as it has in past years. In this instance, extra bridge length should be provided to accommodate this anticipated movement.

In some instances, it may be feasible to change the highway alignment so the crossing will be located in a more stable reach. In other cases, where alignment problems prohibit moving the crossing, stabilization of the banks with riprap may be an economical solution.



# Chapter 5 : HEC 16

## Change in Channel Form with Change in Slope

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Channel modification or straightening of a channel is sometimes necessary to achieve proper highway alignment. As previously mentioned, channels can be in a threshold condition where only a minor disturbance of the channel can cause significant changes.

If the stream is in a threshold condition, a small change in channel slope of a relatively stable, meandering sandbed stream can cause the channel to change to a transitional or braided stream. The latter two forms are generally less stable and less predictable than a meandering channel and additional maintenance can be expected if this change occurs.

If the mean annual discharge and the slope of a sandbed channel are known, Figure 4.4.3, page IV-20, can be used to establish whether the river is near a threshold.

*Example:* If the mean discharge of a meandering channel is 1000 cfs and the slope is 0.0002, a check of Figure 4.4.3 indicates that the river is a meandering river. However, the point plots very close to the limit curve. If the slope must be steepened, or a significant increase in the mean discharge is anticipated, for example with change in land use, the form of the river may change to a transitional or a braided stream. As mentioned above, transitional or braided streams are generally more unstable than meandering streams. This review should be performed for each channel modification when channel shortening is contemplated or when a change in discharge is anticipated.

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# Chapter 6 : HEC 16

## Channel Roughness

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The individual bed elements are the major contributors to roughness in coarse material channels where ripples and dunes are rare and the channel bed is armored.

## Manning's "n" for Large Element Channels

A coarse material channel may have bed material that is partly submerged during most of the flows. The channel roughness cannot be determined for such beds. Where elements are submerged, analysis of data from many rivers, canals and flumes (Anderson, 1968) shows that channel roughness can be predicted by Equation 3.10.46, page III-75:

$$n = 0.04 (D_{50})^{1/6}$$

Where:  $n$  = Manning's roughness coefficient  
 $D_{50}$  = Median diameter (ft).

*Example:* Compute channel roughness for a channel having  $D_{50} = 1.5$  ft.

$$n = 0.04(1.5)^{1/6} = \underline{0.042}$$

The Shields' relation can be used to verify that the bed is not in motion.

## Shields' Relation for Beginning of Motion

The Shields' relation (Figure 3.2.3, page III-5) is useful for predicting the beginning of motion in bed material for given flow conditions. Prior to the beginning of motion, the problem of resistance to flow is one of rigid boundary hydraulics. After beginning of motion, the problem relates to defining bed configuration and resistance to flow. Referring to Figure 3.2.3, for a wide channel, where depth ( $y_o$ ) can be substituted for hydraulic radius ( $R$ ), the parameter

$$\frac{\tau}{(\gamma_s - \gamma)D}$$

can be expressed as

$$\frac{y_o S}{(1.65)D}$$

and the parameter

$$\frac{V_* D}{\nu}$$

can be expressed as

$$\frac{(gy_0 S)^{1/2} D}{\nu}$$

These parameters and Figure 3.2.2 can be used for predicting motion of bed material.

*Example:* Given a wide channel with a flow depth  $y_0 = 4.0$  ft., slope of energy line of the channel  $S = 0.001$ , water temperature =  $70^\circ$  ( $\nu = 1.06 \times 10^{-5}$  ft<sup>2</sup>/sec), the median diameter of the bed material,  $D = 0.01$  ft., and the acceleration of gravity  $g = 32.2$  ft/sec<sup>2</sup>. Will the bed material be in motion or will the bed be stable?

$$\frac{y_0 S}{(1.65) D} = \frac{(4)(0.001)}{(1.65)(0.01)} = \underline{0.24}$$

$$\frac{[(g)(y_0)(S)]^{1/2} D}{\nu} = \frac{[(32.2)(4)(0.001)]^{1/2} (0.01)}{(1.06 \times 10^{-5})} = \underline{338}$$

The point 0.24, 338 plots above the division line in the motion region of Figure 3.2.3 and, therefore, one would expect significant bed material movement in the channel for the given flow conditions.

Note: For values of

$$\frac{(gy_0 S)^{1/2} D}{\nu} \geq 400,$$

the value of

$$\frac{y_0 S}{1.65 D} = 0.048$$

is a constant. The value 0.048 can be used to determine "motion" or "no motion" for large values of

$$\frac{(gy_0 S)^{1/2} D}{\nu}$$

## Manning's "n" for Sandbed Streams

Bed configurations (III-1 to III-22) that may form in an alluvial channel are plane bed without sediment movement, ripples, ripples on dunes, dunes, plane bed with sediment movement, antidunes, and chutes and pools (Figure 3.2.2, page III-3). Different bed configurations may form side by side in a channel of varying depth and/or velocity across a section, may change from reach to reach, and will certainly change with significant changes in discharge or viscosity of the water (temperature change or change in concentration of suspended sediment).

Depending upon the bed form, bed roughness changes considerably as shown in section 3.5.0, page III-20



and III-21. At high flows, most sandbed streams shift from dune to transition or plane bed. The flow resistance then decreases two to threefold with a significant increase in velocity and decrease in stage.

The boundary roughness of a channel must be estimated prior to any hydraulic analysis. The state-of-the-art does not permit a precise estimate of boundary form and consequent roughness for sandbed streams. Therefore, limit values of Manning's "n" are sometimes used for hydraulic computations. The maximum value of "n" from the anticipated range of "n" values should be used when estimating maximum channel stage and evaluating conveyance. When designing riprap or estimating scour depth where velocity is critical, the minimum possible value of "n" should be used.

*Example:* A sandbed stream with  $q = 30$  cfs/ft (design discharge per foot of channel width),  $D_{50} = 0.6$  mm = 0.000183 ft (measured median size of bed material),  $S = 0.0005$  (estimated or measured slope of energy line of the channel), and  $\nu = 0.7 \times 10^{-4}$  ft<sup>2</sup>/sec (estimated or measured).

Figure 3.3.2, page III-15 and Figure 3.3.3, page III-16 are useful for estimating  $v$ .

Determine the maximum anticipated stage and the maximum anticipated velocity for the condition given.

1. Stream power =  $V\gamma S = q\gamma S = (30)(62.4)(0.0005) = 0.9$   
With  $D_{50} = 0.6$  mm and stream power = 0.9, check Figure 3.4.1, page III-20. River is in transition zone; therefore, the bed form is unknown, i.e., the bed form may be dune, plane or antidune and may shift from one to the other.
2. To determine maximum anticipated stage, select maximum value of "n" for possible bed conditions. From page III-21, for dune beds, "n" maximum is 0.04. Combining Manning's equation

$$v = \frac{1.49y^{2/3}S^{1/2}}{n}$$

and the continuity equation  $q = vy$ , yields the equation:

$$y = \left[ \frac{qn}{1.49S^{1/2}} \right]^{0.6}$$

Inserting values for  $q$ ,  $n$  and  $S$ , the maximum anticipated stage is:

$$y = \left[ \frac{(30)(0.04)}{1.49(0.0005)^{1/2}} \right]^{0.6} = \underline{8.6 \text{ ft.}}$$

The velocity associated with this depth can be obtained by the continuity equation

$$v = q/y = \frac{30}{8.6} = \underline{3.5 \text{ ft/sec.}}$$

3. To determine the maximum possible velocity, select the minimum possible value of "n" for possible bed conditions. Figure 3.4.1 page III-20, for plane bed or antidunes, "n" minimum is 0.01. Using equation above, the maximum anticipated velocity is:

$$y = \left[ \frac{qn}{1.49S^{1/2}} \right]^{0.6} = \left[ \frac{(30)(0.01)}{1.49(0.0005)^{1/2}} \right]^{0.6} = \underline{3.74} \text{ ft.}$$

$$v = q/y = \frac{30}{3.74} = \underline{8.0} \text{ ft/sec.}$$

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# Chapter 7 : HEC 16

## Channel Modification

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This section provides design information for those cases where an encroachment requires the replacement of a natural channel with a man-constructed channel.

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## Design of a Meandering Channel with Minor Changes of Slope and Width

The designer should examine similar natural meandering channels in the region and design the proposed channel with similar slope, radii of curvature and channel width. When the slope for the proposed channel must be steeper than the natural channel slope, the proposed channel can be widened such that the stream power in the proposed channel is equivalent to that of relatively stable channels in the vicinity.

For this to be true, Equation 6.2.12, page VI-5 states that

$$W_2 = \frac{S_2}{S_1} W_1.$$

Where:

$W_2$  = Width of proposed channel (ft).

$S_2$  = Design slope of proposed channel (ft/ft).

$S_1$  = Slope of stable meandering channel (ft/ft).

$W_1$  = Width of stable meandering channel (similar discharge, bank material, sinuosity, slope) (ft).

Limitations: Check to see if the proposed channel falls in the meandering stream category:  $W_2 \leq 1.15 W_1$ .

*Example:* Check Figure 4.4.3, page IV-20 for  $S_2 = 0.0004$  and  $Q_{\text{mean}} = 10,000$  cfs.

Channel instability may occur, since the point falls in the intermediate stream zone.

Based on experience in the locality, the designer may elect to stay with the design and live with the possible consequences, i.e., repair and riprap of channel banks if scour does occur and maintenance dredging of the channel if it becomes choked with sediment. Shifts in the location of the thalweg should be anticipated. Bank caving and subsequent deposition in the crossings may cause a shift of thalweg. Maintenance may be expensive

and when changes occur during high flows, the channel will be inaccessible for maintenance.

An alternative is to increase the length of the thalweg thereby decreasing  $S$  such that the channel falls in the meandering stream zone of Figure 4.4.3.

## Design of a Meandering Channel When a Major Shift in Location and Alignment is Necessary

The criteria for this situation are:

1. Slope and formative discharge must be such that  $S$  and  $Q$  plot in the meandering zone of Figure 4.4.3.
2. The meander length ( $\lambda$ ) (see Figure 5.2.1, page V-3 for definition sketch), the mean radius of curvature of the centerline of the channel ( $r_c$ ), and the amplitude of the meander ( $A$ ), must meet the criteria shown on page V-4.

$$\lambda = 10.9 W^{1.01}$$

$$r_c = 2.36 W^{1.03}$$

(This equation is obtained by equating  $\lambda = 10.9 W^{1.01}$  and  $\lambda = 4.7 r_c^{0.98}$  and solving for  $r_c$ )

$$A = 2.7 W^{1.10}$$

Where:  $W$  = Average width of channel at bank-full stage (ft).

Channel width, depth, and slope should be similar to channel width, depth, and slope of similar meandering channels in the region. General relationships derived at CSU, page IV-24\*, for  $W$ ,  $Y$ ,  $S$ , and  $V$  are:

	$Q$ = bank-full discharge (cfs)
$W = aQ^{0.46}$	$W$ = average bank-full width of channel (ft)
$Y = bQ^{0.46}$	$Y$ = average bank-full depth (ft)
$S = cQ^{-0.46}$	$S$ = slope of energy line (ft/ft)
$V = dQ^{0.08}$	$V$ = average velocity (fps)

The coefficients  $a$ ,  $b$ ,  $c$ , and  $d$  must be derived for the physiographic region where the channel change is located.  $W$ ,  $Y$ ,  $S$ ,  $V$ , and  $Q$  for existing stable, meandering streams must be measured or accurately estimated. Then coefficients  $a$ ,  $b$ ,  $c$ , and  $d$  can be computed. With these values of  $a$ ,  $b$ ,  $c$ , and  $d$ , the above formulas are general and can be used for designing channels of any size in similar physiographic regions.

Design Procedure:

1. For the design discharge,  $Q$ , select  $S$  such that  $S$  and  $Q$  plot in the meandering stream zone of Figure 4.4.3, page IV-20.
2. Establish  $W$ .,  $Y$ ., and  $S$  using regional data, i.e., data from existing channels.



- Determine  $\lambda$ ,  $r_c$ , and  $A$  and prepare plan of channel change using proper values of  $S$ ,  $W$ ,  $Y$ ,  $\lambda$ ,  $r_c$ , and  $A$ .

\*Note: Equations shown on page IV-24 are in error. Equal signs must be replaced with proportional signs.

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## Design of a Straight, Steep Channel

Where it is necessary to have a straight, steep channel, there are two alternatives:

- Line the entire length of channel, construct an energy dissipating structure at the downstream end of the reach and construct a transition to the existing channel. Established procedures are available for this alternative.<sup>3,4</sup>
- Subdivide the steep channel into a series of short stable reaches and use drop structures to absorb a portion of the excess energy at each channel drop.

For the design of alternate 2, a suggested method is:

Determine  $D_{50}$  of bed material, design discharge, normal flow depth, specific gravity of bed material, factor of safety, centerline slope, and length of reach.

Using Shields' criteria and a suitable factor of safety, compute the maximum permissible slope of the stream. For given normal flow depth and large Reynolds numbers, information shown on Figure 3.2.3, page III-5 indicates that,

$$\frac{y_o S}{(S_p Gr - 1.0) D_{50}} = 0.048$$

The permissible slope is, therefore:

$$S = \frac{0.048 (S_p Gr - 1.0) (D_{50})}{(\text{Factor of Safety}) (y_o)}$$

*Example:*  $D_{50} = \frac{1}{2}$  in = 0.04 ft; centerline slope of proposed channel,  $S = 0.002$ ; factor of safety = 2.0; length of reach = 4000 ft normal flow depth,  $y_o = 3.0$  ft.; and, specific gravity of bed material is 2.65.

Therefore, the allowable slope of the channel between drop structure is:

$$S = \frac{(0.048)(1.65)(0.04)}{(2)(3)} = \underline{0.00053},$$

and the amount of channel drop = 4000 ft x 0.002 = 8.00 ft.

The drop to be absorbed by structures is: 8.00 - (0.00053) (4000) = 5.9 ft.

Three drop structures, equally spaced with a drop of about 2 ft each could be used to absorb the 5.9 ft. A centerline profile of the proposed design is shown in [Figure 4](#).

If rock riprap drop structures are used, the longitudinal slope of the spillway of the drop structure should not exceed about 7 to 1. Rock should be generously sized as velocity will often range from 10 to 20 fps down the face of the drop structure, depending upon the drop.

Methods for sizing the rock are not well defined; however, rock equal to or larger than the size suggested by the Bureau of Reclamation curve titled "Small Stilling Basins", Figure 3.9.2, page III-40, is suitable. As an example, for velocities down the drop structure face of 12 fps the diameter of the stone should be at least 1.8 ft.

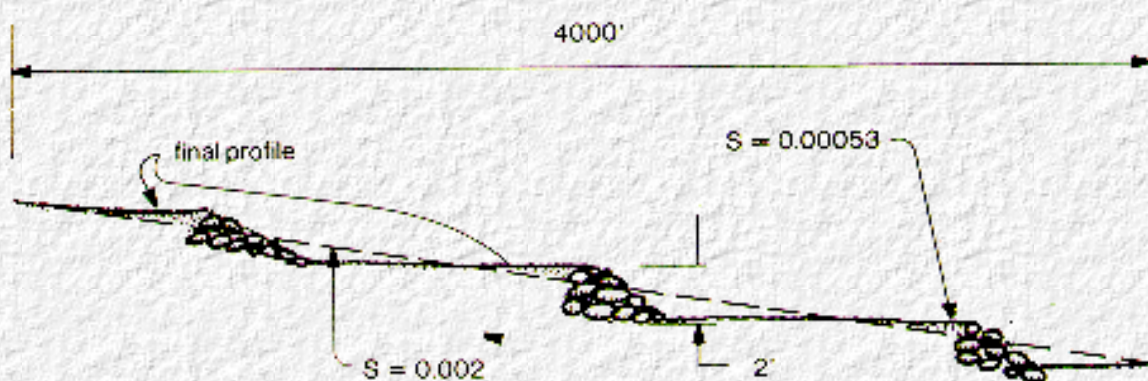


Figure 4. Centerline Profile of Proposed Design (No Scale)

Rock size, geometry of the structure, proper riprap of channel banks in the vicinity of the drop structure, careful rock placement, and proper filter design are all of equal importance for a satisfactory structure. Although detailed specifications for drop structures are beyond the scope of this addendum, information concerning design of filters, gradation of riprap and placement of riprap (VIA-1 through VIA-33) are applicable.

<sup>3</sup> "Hydraulic Design of Energy Dissipators for Culverts and Channels", HEC No. 14, U.S. Department of Transportation, FHWA, 1975.

<sup>4</sup> "Design of Small Dams", U.S. Department of the Interior, Bureau of Reclamation, 1973.

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# Chapter 8 : HEC 16

## Bends in Alluvial Channels

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An estimate of scour depth at a bend in a channel is necessary for establishing the toe elevation of riprap or elevation of pier footings located within the bend. Bend depth is defined as the average depth of flow in the cross section. Scour depth is the maximum depth, measured from the water surface, which can be expected in the bend.

## Classification of Bends

Bends can be classified in the following manner (V-7 to V-8):

*Free bends.* Both banks composed of alluvial flood plain material which is usually quite mobile,

$$r_c/W_f = 4.5 \text{ to } 5.0.$$

*Limited bends.* The banks are composed of consolidated parent material which limits the intensive development of lateral erosion by the stream (an entrenched bend),

$$r_c/W_f = 7.0 \text{ to } 8.0.$$

*Forced bends.* The stream impinges onto an almost straight consolidated parent bank at a large angle (60°-90°).

$$r_c/W_f = 2.5 \text{ to } 3.0.$$

Where:  $r_c$  = radius of channel centerline.

$W_f$  = top width of channel at bank-full discharge.

In the first two types of bends, the depth gradually increases and the maximum depth is found some distance below the apex of the bend. In the forced bend, the depth sharply increases at the beginning of the bend and then gradually diminishes. The greatest depth is located in the middle third of the bend.

## Estimating Bend Depth

*Regional Method.* In the immediate vicinity, locate several similar channels with similar alignments flowing in similar material and measure the depths and lengths of pools. From this information, anticipated scour depths can be estimated for the proposed channel.

*Russian Method*<sup>5</sup> (V-16 to V-25). The following limitations apply to this method:

1. Can only be used for rivers with low mean annual sediment concentrations - maximum concentrations up to 90 ppm.
2. Design parameters are available only for stream orders ranging from IV to XIV.

3. The index of stability (see below) of the channel must fall within the range of 0.5 to 1.5.

Stream order is defined as shown in [Figure 5](#). The elementary unbranched stream is a first (I) order stream. Two first order streams combine to form a second (II) order stream, etc.

The first (I) order stream is defined as an intermittent watercourse where the direction of flow down the slope is determined by a gully, ravine, or valley. The channel is shaped only during periods when large quantities of water enter along the channel. During the rest of the year, the watercourse is dry.

The index of stability (x) is defined as:

$$x = \frac{D_{50} W}{y_o^2 S}$$

Where:  $D_{50}$  = median diameter of bed material (ft).  
 $W$  = average width of stream (ft).  
 $y_o$  = average depth of flow (ft).  
 $S$  = slope of the energy grade line (ft/ft).

*Example:*

Estimated mean annual sediment concentration = 80 ppm,  $D_{50}$  = 0.001 ft.  $W$  = 220 ft,  $y_o$  = 8 ft.  $S$  = 0.0025,  $r_c$  = 1760 ft. and stream order = IV (Determined by examining topographical map).



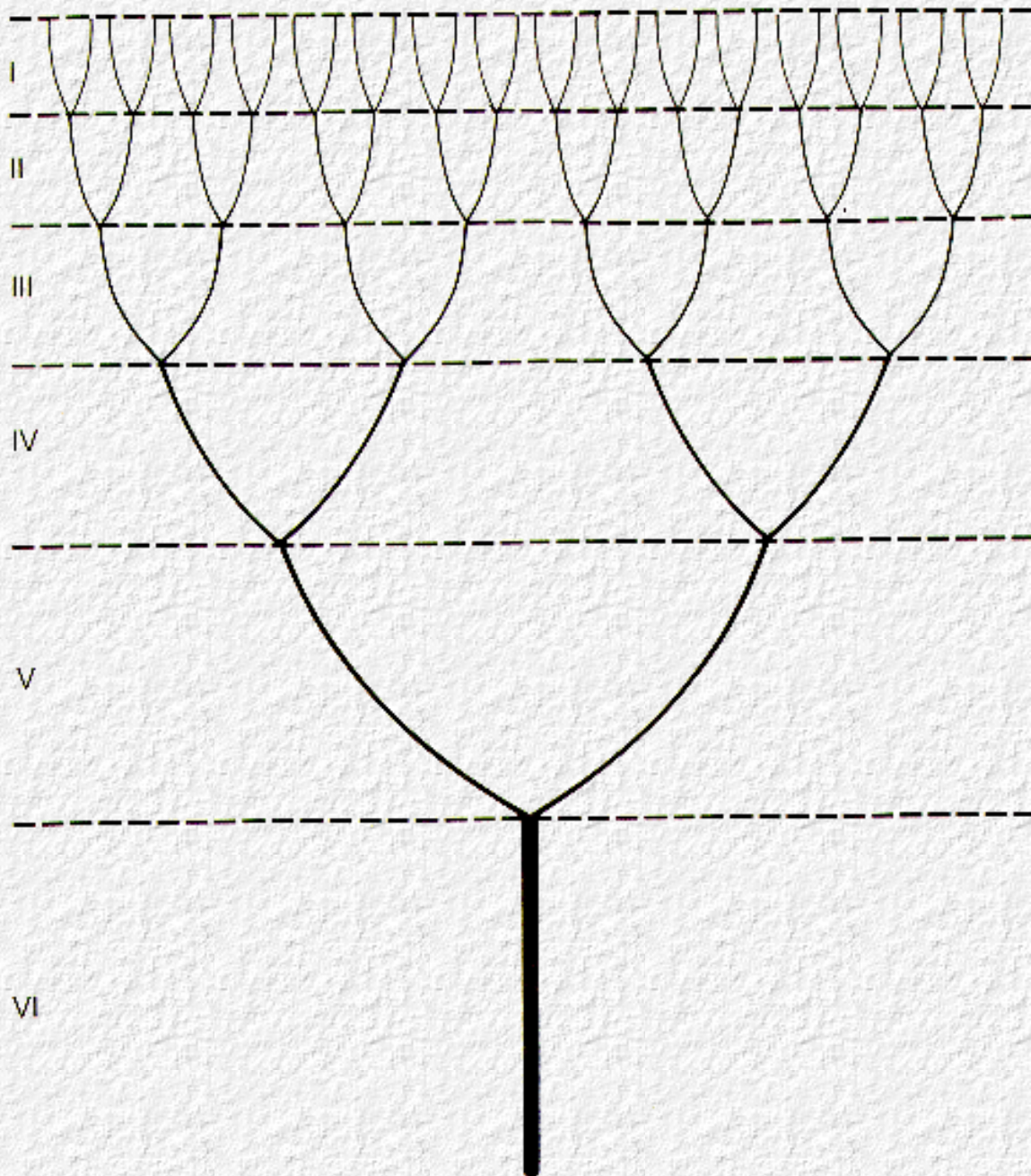


Figure 5. Scheme of a River Net of the VI Order. Numerals Indicate Stream Orders.  
(From "Morphological and Hydrological Regularities of the Structure of the River Net")

*Procedure:*

$$x = \frac{D_{50} W}{y_0^2 S} = \frac{(0.001)(220)}{(8)^2 (0.0025)} = \underline{1.4}$$

$$\frac{r_c}{W} = \frac{1760}{220} = \underline{8}$$

Stream order = IV

From Figure 5.6.3, page V-20,

$$\frac{y_{\max}}{W} = 0.047; \text{ therefore,}$$

$$Y_{\max} = (220)(0.047) = \underline{10.34} \text{ ft.}$$

*Comments:*

1. Most streams will carry a mean annual sediment load significantly greater than 90 ppm.
2.  $\chi$  for many streams will not fall within the range of 0.5 to 1.5.
3. If the river does meet the above criteria, additional information concerning length of pools and location of points of maximum scour can be obtained from information shown on pages V-20 through V-23.

*Blench Method*

This method is not discussed in HEDC. The method is limited to gravel and cobble-bed channels where the banks are riprapped at the time the channel is constructed.

Blench<sup>6</sup> noted that, "The depth obtained is a practical guide for estimating the scoured depth to be used for estimating the toe elevation but not to obtain the scour depth that might actually occur without the riprap." Neill<sup>7</sup> reporting on a case study of eleven scour holes at sharp bends along a stretch of the Athabasca River in Alberta, Canada, found good agreement between measured and computed bend depths. Neill used the 10-year flood as the channel forming discharge. The size of bed material ( $D_{50}$ ) was 1-3/4 inches and  $D_{90}$  was about 3 inches.

The formulas recommended as a practical guide for estimating design scour depths are:

$$d_s = C \frac{(q_f)^{2/3}}{(F_{bo})^{1/3}}$$

Where:  $q_f = \frac{\text{channel forming discharge}}{\text{mean channel width}}$

$$F_{bo} = 7.3(D_{50})^{1/4}$$

$D_{50}$  = mean diameter of bed material by weight (ft).

$C = 1.7$  at severest point for a free bend, and 2.0 to 2.9 for abrupt impingement of flow on a long bank (forced bend).

$d_s$  = total depth from water surface to bottom of scour hole (ft)

The channel forming discharge is often taken as the bank-full discharge.

Mean channel width is the average of bank-full top width and low-water top width.

*Example:* Estimate the maximum anticipated scour depth in a riprapped, moderate bend of a channelized section.

For a moderate bend,  $C = 2.0$ . The mean surface width = 512 ft,  $D_{50} = 0.25$  ft. and formative discharge,  $Q_{10} = 100,000$  cfs.



$$F_{b0} = 7.3(0.50)^{1/4} = (7.3)(0.25)^{1/4} = \underline{5.16}$$

$$q_f = \frac{100,000}{512} = 195.3 \text{ cfs/ft}$$

$$d_s = C \frac{(q_f)^{2/3}}{(F_{b0})^{1/3}} = \frac{(2.0)(195.3)^{2/3}}{(5.16)^{1/3}} = \underline{39.0 \text{ ft}}$$

The maximum anticipated depth from the water surface to the bottom of the scour hole is 39.0 ft.

<sup>5</sup>N. A. Rzhanitsyn, "Morphological and Hydrological Regularities of the Structure of the River Net", translated by D. B. Krimgold, U.S. Department of Agriculture, Agricultural Research Service, Soil and Water Conservation Research Division, 1960.

<sup>6</sup>T. Blench, "Mobile-Bed Fluviology," The University of Alberta Press, Edmonton, Alberta, Canada, 1969.

<sup>7</sup>C. R. Neill, "Scour Holes in a Wandering Gravel River," Rivers 76 Published by ASCE, 345 East 47th Street, New York, New York, 1976.

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# Chapter 9 : HEC 16

## Sediment Transport - Bed Load

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Because sediment transport is a complex topic, the results generated from analysis are typically limited to order of magnitude estimates. The hydraulic engineer should study Chapter III in its entirety to become familiar with the principles involved. A brief review of the use of three methods, Colby's, Einstein's and Meyer-Peter Muller's, is presented here.

## Colby's Method for Computing Bed Material Discharge

This method (III-69 - III-71) is applicable only to sandbed streams,  $0.1 \text{ mm} < D_{50} < 0.8 \text{ mm}$

The data required are:

1. Water temperature.
2. Fine sediment concentration in ppm.
3. Characteristics of average cross section (perimeter of bed, average area, and hydraulic radius as a function of depth, Figure 3.10.9, page III-63 and table 3.10.3, page III-65).
4. Appropriate range of discharges and associated normal depths.
5. Gradation of average bed material (table 3.10.2, page III-64).

Computation Procedures:

Two methods are suggested, (pages III-69 and III-71).

Method 1: Computations based on using the median diameter of bed material.

An explanation of the calculations and results is included on page III-69, HEDC.  $Q_T$ , column 9, Table 3.10.5, page III-69, lists the total bed material discharge for the entire cross section for the specific normal depth shown in column 1. As an example, for  $y_o = 4.14 \text{ ft}$ ,  $Q_T = 85,410 \text{ tons/day}$ .

Method 2: Computations using individual size fractions of bed material.

Table 3.10.6, page III-70, lists the bed-material discharge ( $i_B Q_T$  column 12) for the specified size fraction (column 1) and specified flow depth (column 3) for the entire cross section. To obtain the total bed material discharge at the cross section for the entire range of size fractions, the four values of  $i_B Q_T$  associated with the specified value of  $y_o$  must be summed.

*Example:* For  $y_o = 4.14 \text{ ft}$ .  $Q_T = 11,900 + 30,420 + 29,000 + 6,320 = 77,640 \text{ tons/day}$ .

This value can be compared to the value obtained above (method 1) where  $Q_T = 85,410 \text{ tons/day}$ .

Colby's Method is the simplest of the three methods for estimating bed-material discharge for sand



bed streams. The discharge-of-sand relationships are based on a large amount of data from streams and flumes. Einstein's load function was used as a guide for extrapolating and shaping the curves (Figure 3.10.10, page III-67 and Figure 3.10.11, page III-68). When checking the design curves against measured data that was used for developing the curves, Colby found that 75 percent of the measured sand discharges were less than twice or more than half of the discharges than were estimated using the graphs.

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# Einstein's Method for Computing Total Bed-Material Discharge

This method (III-63 through III-66) has the following limitations:

- 1. Primarily useful for sand bed streams where there is a limited amount of sediment carried in suspension.
- 2. Although the procedure is theoretically sound, it is somewhat cumbersome and time consuming without the use of a computer.

Data Required:

- 1. Cross section parameters as a function of depth (Figure 3.10.9, page III-63).
- 2. Grain size distribution of bed material (Table 3.10.2, page III-64). Most computer programs will accept  $D_{50}$  and the gradation coefficient.
- 3. Viscosity of water sediment mixture flowing in channel.
- 4. Stage discharge relation for the reach.

Computation Procedures:

See Tables 3.10.2, 3.10.3 and 3.10.4, pages III-64 through III-66\* for typical computations and explanations of each step.

Results:

In table 3.10.4 page III-66, the total bed-material load, contact load plus suspended sediment in the upper layer of the bed, is given in column 19.

The total load for all size fractions for any bed hydraulic roughness,  $R_b''$ , is the sum of the loads for each size fraction.

Example:  $R_b'' = 4.0$

$D = .00162$	Total load,	$i_T Q_T = 12,100$ tons/day
$D = .00115$		$i_T Q_T = 57,100$
$D = .00081$		$i_T Q_T = 157,000$
$D = .00057$		$i_T Q_T = 97,800$

Total bed material for all size fractions = 324,000 tons/day

A comparison of water discharge versus bed-material load computed by Colby's method and Einstein's method is shown in Figure 3.10.12, page III-71.

Einstein's method is based on observed bed load material measurements and is appropriate for computing bed load where most of the material moves as bed load rather than in suspension. Colby's curves present a practical alternative method for checking bed load computations from the Einstein method or for estimating bed load when the scope of the project does not warrant an expensive sediment transport analysis. It is noted in HEDC page III-64, ". . . that all curves (Colby's) for 100 ft depth, most curves of 10 ft depth and part of the curves of 1.0 ft and 0.1 ft are not based on data but are extrapolated from limited data and theory."

\*Notes:

1. In table 3.10.2, the average grain size in millimeters is the geometric mean of the particle size range.

*Example:* for  $0.417 < D < 0.589$ ,  
 $D = [(0.589)(0.417)]^{1/2} = 0.495$

2. Typographical errors in HEDC, Table 3.10.3 page III-65

footnote (3)  $\phi' = 11.62 \sqrt{V_*'}$  (V\*' instead of U\*')

footnote (8)  $\psi' = \frac{\rho_s - \rho}{\rho} \frac{D_{35}}{R_b' S}$  (D<sub>35</sub> instead of D<sub>65</sub>)

Table 3.10.4, page III-66

footnote (4)  $\bar{\psi}' = \frac{\rho_s - \rho}{\rho} \frac{D}{R_b' S}$  (D instead of d)

footnote (13)  $Z = \omega / 0.4 \sqrt{V_*'}$  (V\*' instead of U\*')

## Meyer-Peter and Muller Equation for Computing Bed-Material Load

This method (III-52 through III-55 and IIIA-1 through IIIA-4) is recommended for estimating coarse bed material transport.

For data required and results see pages IIIA-1 through III-4.\*

The Meyer-Peter Muller equation is the only tested equation for estimating bed material discharge for coarse material ranging up to 33 mm in size. There is no known method for estimating sediment transport for cobble-boulder streams.

\*Note:



Typographical error line 9, page IIIA-4, parentheses missing in term  $\left(1 - \left(\frac{0.060}{0.040}\right)^{3/2}\right)$ .

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# Chapter 10 : HEC 16

## Scour

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Three types of interrelated phenomena cause changes in bed level at crossings or encroachments. They are:

1. Local scour at the base of piers, dikes, or other obstructions that create vortices and eddies in the stream.
2. General scour resulting from contractions - examples of contracted stream channels include narrow channels created by the use of embankments, and accumulation of debris at bridge openings.
3. Long term degradation or aggradation of a channel. This can result from changes in sediment load, change in flow regimes or where the river is seeking an equilibrium slope.

Items 2 and 3 above are beyond the scope of this addendum. The reader is referred to chapters IV, V, and VI for information on these types of scour. An outline of computation procedures for specific cases of local scour is presented below.

### Case A. Estimate maximum depth of scour in a sandbed stream at a spill slope when the flow is subcritical (VI-32 to VI-42).

Data Required:

$y_1$  = normal flow depth for design discharge just upstream of the crossing (ft).

$V_1$  = average velocity for design discharge just upstream of crossing (ft/sec).

$a$  = embankment length - perpendicular distance from the high water line at the valley bank to the end of the embankment at the bridge (ft).

Modifying Equation 6.5.6, page VI-35 to:

Where:  $y_m$  = maximum depth of scour measured from the mean bed level to the bottom of the scour hole. This value is 30% greater than equilibrium scour, ( $y_m = 1.3 y_s$ ), and

$$F_{r1} = \frac{V_1}{[(32.2)(y_1)]^{1/2}} \quad \text{The upstream Froude Number.}$$

*Example:* In this case, the following conditions must exist: sandbed stream, subcritical flow,



scour at spill slope toe, embankment perpendicular to valley banks, and  $0 < a/y_1 < 25$ .

$a = 200$  ft,  $y_1 = 12$  ft,  $V_1 = 8$  fps

$a/y_1 = 200/12 = \underline{16.7}$   $0 < a/y_1 < 25$ , o.k.

$$F_{r1} = \frac{V_1}{(gy_1)^{1/2}} = \frac{8}{(32.2 \times 12)^{1/2}} = \underline{0.41} \quad (\text{subcritical}), \text{ o.k.}$$

$$y_m = y_1 [1.43(a/y_1)^{0.4} F_{r1}^{0.33}]$$

$$y_m = 12 [1.43(200/12)^{0.4} (0.41)^{0.33}] = \underline{39} \text{ ft}$$

---

## Case B. Estimate maximum scour depth in a sandbed stream where the embankment terminates at a vertical wall and has a vertical wall on the upstream side.

Modifying equation 6.5.8 (page VI-35) to:

$$y_m = 2.8 y_1 [(a/y_1)^{0.4} F_{r1}^{0.33}].$$

In this case, the following conditions must exist: sandbed stream, subcritical flow, scour at toe of vertical wall where vertical wall exists both upstream and along the face of the embankment, embankment perpendicular to valley wall, and  $0 < a/y_1 < 25$ .

Variables as defined in case A.

$a = 200$  ft,  $y_1 = 12$  ft,  $V_1 = 8$  fps

$a/y_1 = 200/12 = \underline{16.7}$   $0 < a/y_1 < 25$ , o.k.

$$F_{r1} = \frac{V_1}{(gy_1)^{1/2}} = \frac{8}{(32.2 \times 12)^{1/2}} = \underline{0.41} \quad (\text{subcritical}), \text{ o.k.}$$

$$y_m = (2.8)(12)[16.7^{0.4} (0.41)^{0.33}] = \underline{77.2} \text{ ft}$$

---

## Case C. Estimate maximum scour depth in a sandbed stream at the end of a rock dike.

Modifying equation 6.5.9, page VI-35 to:

$$y_m = 5.2 y_1 F_{r1}^{0.33}.$$

In this case the following conditions must exist: sandbed stream, subcritical flow, and  $a/y_1 > 25$ .

$a = 400$  ft,  $y_1 = 12$  ft,  $V_1 = 8$  fps

$a/y_1 = 400/12 = \underline{33.3}$   $a/y_1 > 25$ , o.k.

$$F_{r1} = \frac{v_1}{(gy_1)^{1/2}} = \frac{8}{(32.2 \times 12)^{1/2}} = \underline{0.41} \text{ (subcritical), o.k.}$$

$$y_m = 5.2(12)[F_{r1}]^{0.33}$$

$$y_m = (5.2)(12)(0.41)^{0.33} = \underline{46} \text{ ft}$$

## Case D. Estimate the maximum local scour where the embankment is constructed at an angle to the valley.

If the embankment is angled downstream, the depth of scour is reduced because of the streamlining effect. Embankments angled upstream have deeper scour holes.

Figure 6.5.8, page VI-37, suggests a correction factor for modifying scour depth estimates for this condition.

In this case the following conditions must exist: the embankment is constructed at an angle other than 90° (e.g., angled downstream) to the valley, sandbed stream, subcritical flow, and  $a/y_1 < 25$ .

Variables as defined in Case A, above except embankment is angled 30° downstream from perpendicular to bank.

Angle of inclination = 90° - 30° = 60°.

Ratio of depth of scour to that for a normal embankment = 0.94 (Figure 6.5.8, page VI-37).

Anticipated maximum scour depth is (0.94)(39) = 37 ft

## Case E. For flow conditions as in Case A, estimate the local scour around a square-nosed pier aligned with the flow.

Maximum scour is 30% larger than equilibrium scour depth.

Modifying equation 6.5.10, page VI-38 to:

$$y_m = 2.86 (y_1) (a/y_1)^{0.65} (F_{r1})^{0.43}.$$

Where: a = width of pier.

Example: a = 2.00 ft.  $F_{r1} = 0.41$ ,  $y_1 = 12.0$  ft

$$y_m = 2.86(12.0)(2/12)^{0.65} (0.41)^{0.43} = \underline{7.3} \text{ ft}$$

For pier noses other than square-nosed, see correction factor, table 6.5.1, page VI-39.

For piers that are not aligned with flow, see correction factor, table 6.5.2, page VI-40.







# Chapter 11 : HEC 16

## Safety Factor for Riprap

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A method for reviewing the safety factor of riprap is described in this section. Only the form of the equations using mean flow velocity and flow depth (parameters used in other sections of this addendum) are reviewed. Other forms of these equations are illustrated in chapters VI and VIII.

### Determine the Safety Factor for Bank Riprap Placed along a Straight Channel

The procedures for determining the safety factor for this case is best illustrated by an example: Channel side slopes 2:1; average velocity,  $V = 10$  fps; average flow depth,  $y_o = 8.0$  ft; selected size of riprap,  $D = 1.0$  ft; riprap is very angular.

Procedure:

1. Reduce the side slope to an angle and determine the value of  $\sec \theta$   
 $\theta = \tan^{-1} (1/2)$ ;  $\theta = 26.6^\circ$ ;  $\sec \theta = 1.118$
2. Determine the angle of repose for very angular riprap,  $D = 12$  inches.  
 From Figure 3.7.3, page III-29, angle of repose,  $\Phi = 42^\circ$
3. Obtain reference velocity ( $V_r$ ) using Equation 6.A3.7, page VIA-18.

$$V_r = V \frac{3.4}{\ln (12.3 y_o / D)} = \frac{(10) (3.4)}{\ln (12.3 \times 8 / 1)} = \underline{7.4 \text{ fps}}$$

4. Obtain the safety factor for rock riprap on side slopes where the flow is horizontal (Equation 6.4.11, page VI-22).

$$\text{S.F.} = S_m / 2 [(\epsilon^2 + 4)^{1/2} - \epsilon]$$

$$\text{Where } \epsilon = S_m \eta \sec \theta$$

$$S_m = \frac{\tan \phi}{\tan \theta} = \frac{\tan 42^\circ}{\tan 26.6^\circ} = 1.798$$

(Equation 6.4.10, page VI-22)

$$\eta = \frac{0.3 V_r^2}{(S_s - 1) g D} = \frac{(0.3)(7.4)^2}{(1.65)(32.2)(1)} = \underline{0.309}$$

(Equation 6.A3.5, page VIA-18)

$$S_s = \text{Specific gravity of riprap} = \underline{2.65}$$

$$\text{Compute: } \epsilon = (1.798)(0.309)(1.118) = \underline{0.621}, \text{ and}$$

$$\text{S.F.} = 1.798 / 2 [(0.621^2 + 4)^{1/2} - 0.621] = \underline{1.32}$$



If the factor of safety is too small (1.5 is suggested, page VIA-25) the size of rock, D, can be adjusted upward and/or the side slope angle can be reduced. Various trial combinations can be reviewed until a satisfactory safety factor is obtained.

---

## Riprap Size If Safety Factor Is Specified

The same data as for Case A, except rock size D is not specified and a safety factor of 1.5 is used.

$$\phi = 42^\circ, \theta = 26.6^\circ, V = 10 \text{ fps}, V_r = 7.4 \text{ fps}, S_s = 2.65$$

Procedure from Figure 6.A5.1, page VIA-26,

Stability factor, = 0.19.

Combining equations 6.4.5, page VI-21 and equation 6.A3.4, page VIA-17 by equating  $T_o'$  and solving for D:

$$D = \frac{0.3V_r^2}{(S_s - 1)g_n} = \frac{0.3(7.4)^2}{(1.65)(32.2)(0.19)} = \underline{1.6} \text{ ft}$$

Rock size is very sensitive to side slope angle. If a side slope of 3:1 ( $\theta = 18.43^\circ$ ) is used in the above example, the required rock size would be 0.73 ft as compared to 1.6 ft for a side slope of 2:1.

---

## Determine the Safety Factor for Riprap at the Toe of an Embankment (VIA-39)

See Figure 6.A8.2, page VIA-39 for point identified as "Toe of Embankment". For this case the conditions are:

Example: Side slope 2:1 ( $\theta = 26.6^\circ$ ), very angular riprap,  $D = 1.3 \text{ ft}$ .  $\phi = 42^\circ$ , average velocity in channel upstream of embankment,  $V_1 = 10 \text{ fps}$ , average upstream flow depth for maximum backwater associated with design, discharge,  $h_1 = 8.0 \text{ ft}$ . total drop in the water surface elevation as the flow passes through the bridge opening,  $\Delta h = 0.7 \text{ ft}$ . and the velocity coefficient,  $\alpha = 1.25$  (obtain  $\Delta h$  and  $\alpha$  using procedures recommended in reference 8).

Nonuniform flow is defined as the condition where flow depth for a constant discharge varies from section to section along the centerline of a channel. Backwater is defined as a condition of nonuniform flow where the depth of flow exceeds uniform flow depth for a specified discharge (see pages II-44 through II-51). At most bridges where an encroachment occurs, backwater will occur and it will be necessary to estimate the water surface profile. The reader is referred to the publication cited above<sup>8</sup> for additional details on nonuniform water surface profiles at highway crossings.

Procedure:

1. Flow is essentially horizontal and parallel, therefore  $\lambda$ , the angle between the velocity vector and a horizontal plane, is zero.
2. Compute  $V_d$ , the average velocity through the contracted section of the bridge opening, by Equation 6.A8.1, page VIA-40.

$$V_d = (\alpha_1 V_1^2 + 2(g) \Delta h)^{1/2} = ((1.25)(10)^2 + 2(32.2)(0.7))^{1/2}$$

$$V_d = \underline{13.0 \text{ fps}}$$

3. Compute  $V_t$  from information shown on figure 6.A8.3, page VIA-41.

$$\text{For } \Delta h/L = 0.7/45 = 0.0156,$$

$$V_t/V_d = 0.93; \text{ therefore, } V_t = (0.93)(13.0) = \underline{12.1 \text{ fps.}}$$

4. Solve equation 6.A3.7, page VIA-18 for  $V_r$  by letting  $V = V_t$

$$V_r = \frac{V(3.4)}{\ln \left( \frac{12.3 y_o}{0} \right)} = \frac{(12.1)(3.4)}{\ln \left( \frac{12.3 \times 8.0}{1.3} \right)} = \underline{9.5 \text{ fps}}$$

5. Compute Safety Factor:

$$S.F. = S_m/2 [(\epsilon^2 + 4)^{1/2} - \epsilon]$$

$$S_m = \frac{\tan \phi}{\tan \theta} = \frac{\tan 42^\circ}{\tan 26.6^\circ} = \underline{1.798}$$

$$\eta = \frac{0.3 V_r^2}{(S_s - 1)(32.2)(0)} = \frac{(0.3)(9.5)^2}{(1.65)(32.2)(1.3)} = \underline{0.392}$$

$$\epsilon = S_m \eta \sec \theta = (1.798)(0.392)(1.118) = \underline{0.788}$$

$$S.F. = 1.798/2 [(0.788^2 + 4)^{1/2} - 0.788] = \underline{1.232.}$$

<sup>8</sup>Federal Highway Administration, 1970, Hydraulics of Bridge Waterways: Hydraulics Branch, Bridge Division, Offices of Engineering and Operation, Hydraulics Design Series, No. 1, 2nd Edition, U.S. Government Printing Office, Washington, D.C.

## Determine the Safety Factor for Riprap at the Flow Separation Point of an Embankment

The anchor point for velocity vector,  $V_p$ , is shown on figure 6.A8.2, page VIA-39.

The data for this case is the same as above.

Procedure:

1. Estimate angle  $\lambda$ , the angle between the velocity vector,  $V_p$ , and a horizontal plane.

$$\Delta h/L = 0.7/45 = 0.0156$$



From Figure 6.A8.4, page VIA-41,  $\Delta h/L = 0.0156$ ,  $\lambda = 13^\circ$ .

2. Compute  $V_r$ : From Figure 6.A8.3, page VIA-41,  $\Delta h/L = 0.0156$ ,  $V_p/V_D = 0.82$ , and  $V_p = (0.82)(13.0) = \underline{10.7}$  fps.

The local flow depth (depth from the surface to the point being reviewed) in this case is

$\frac{h_1}{2}$ ; therefore,

$$y_o = h_1/2 = 8/2 = \underline{4.0} \text{ ft}$$

Letting  $V = V_p$  and solving Equation 6.A3.7, page VIA-18 for  $V_r$  gives:

$$V_r = \frac{V_p (3.4)}{\ln (12.3 y_o/D)} = \frac{(10.7) (3.4)}{\ln (12.3 \times \frac{4.0}{1.3})} = 10.0 \text{ fps}$$

3. Compute factor of safety from Equation 6.A1.30, page VIA-8.

$$S.F. = \frac{\cos \theta \tan \theta}{n' \tan \phi + \sin \theta \cos \beta}$$

Solve equation 6.A1.29, page VIA-8 for

$$\beta = \tan^{-1} \frac{\cos \lambda}{\frac{2 \sin \theta}{n \tan \phi} + \sin \lambda}$$

$$n = \frac{0.3 V_r^2}{(S_s - 1) g D}$$

(Equation 6.A3.5, page VIA-20)

$$n' = n \left( \frac{1 + \sin (\lambda + \beta)}{2} \right)$$

(Equation 6.A1.33, page VIA -8)

Therefore:

$$n = \frac{(0.3)(10.0)^2}{(1.65)(32.2)(1.3)} = 0.434,$$

$$\beta = \tan^{-1} \left( \frac{\cos 13^\circ}{\frac{2 \sin 26.6^\circ}{0.434 (\tan 42^\circ)} + \sin 13^\circ} \right) = 21.2^\circ,$$

$$n' = (0.434) \left[ \frac{1 + \sin (13^\circ + 21.2^\circ)}{2} \right] = 0.339, \text{ and}$$

$$\text{S.F.} = \frac{\cos 26.6^\circ \tan 42^\circ}{(0.339)(\tan 42^\circ) + (\sin 26.6^\circ)(\cos 21.2^\circ)} = \underline{1.1}$$

The region around the embankment nose requiring the calculated size of riprap protection is shown in Figure 6.A8.2, page VIA-39. The angle varies according to information presented in Table 6.A8.1, page VIA-42. The other region of the embankment may be protected by a smaller size of riprap.

Figure 6.A8.3 and Figure 6.A8.4 were developed from model data for spill-through embankments normal to the flow and require modifications when applied to other types of embankments. The writer is unaware of any prototype evaluations of this procedure. However, the procedure is based on a logical method and there is no other known method of checking riprap stability in an accelerating flow zone.

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# Chapter 12 : HEC 16

## Summary

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The complexity of the problems and uncertainties associated with hydrologic, geomorphic and other data precludes the establishment of a standard review procedure for river crossings or encroachments. Depending upon the location, the scope of the project, the design life of the crossing, and source of funding, the amount of design effort may vary considerably. In some cases, formal computations will not be required. In other cases, several man-years of analysis by a team of specialists will be required before a final design is prepared. A construction foreman or supervisor may input to the design based on knowledge and experience with rivers in the region.

When an engineering analysis is required, the procedure outlined below is suggested.

### Qualitative Analyses

1. Document the history of the stream activity and stability.
2. Classify the stream.
3. Generate the significant hydrologic and hydraulic data. This data may have to be upgraded or modified as various trial designs are reviewed, i.e., hydraulic characteristics of the channel will depend on the modified cross section.
4. Review the stability of the original and modified channel.

### Quantitative Analysis

5. Design components of the project as required - riprap, channel training structures, footing depths for piers, etc.

Chapter VIII of HEDC provides excellent background material and outlines some suggested techniques for analyzing site-specific problems. Some problems normally encountered at crossings and encroachments are outlined and reviewed, pages VIII-1 through VIII-22. Actual case histories, countermeasures which were used, and results are reviewed, pages VIII-22 through VIII-37.

Principle factors to be considered in design, such as location of crossing, classification of rivers, river geometry, hydrologic data, hydraulic data, characteristics of watershed, local flow alignment, and flood plain flow are discussed, pages VIII-40 through VIII-45. Site selection, channel stability investigations, short-term response, and long-term response are discussed, pages VIII-45 through VIII-47.

Four illustrative design examples are given starting with page VIII-47. Significant data, selection of Q-dominant and general classifications and history of river are discussed, pages VIII-47 through VIII-52.

The problems are self-explanatory with step-by-step discussion. The logic used for selecting appropriate formulas and specific computations are presented in a clear and concise manner. Some uses of sediment transport relationships are shown on page VIII-77 and design example 4, page VIII-78. The remainder of the chapter deals with analysis of the site-specific problems and possible methods of analysis.

The complexity of river mechanics problems prevents the use of a simple string of design formulas or a standard computation sheet with a specific bottom-line answer. Professional judgment is required throughout the process. The numerical answer obtained must be measured against previous observations and current state-of-the-art limitations. A reasoned decision then leads to the next step of analysis.

The HEDC and this addendum were developed to stimulate hydraulic engineers to utilize the most recent and appropriate tools that are available in the interest of designing trouble-free crossings and encroachments.

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